Pearl Condominiums

9th & Arch Street
Philadelphia, PA

AE Senior Thesis

Final Report

April 9, 2008

Faculty Advisor:
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Joseph G. Lichman Jr.   Structural Option

http://www.engr.psu.edu/ae/thesis/portfolios/2008/jgl138/

Pearl Condominiums
9th and Arch Street
Philadelphia, PA

The Project Team:

Owner - Parkway Corporation
Architect – Blackney Hayes Architects
Construction Manager – JJ Deluca Company Inc.
Structural Engineer – Pennoni Associates Inc.
Civil Engineer – Valimer Associates, LLP
Mechanical Engineer – M.P. Hershman, PE, Inc.
Electrical Engineer – DGW Electrical Engineers, Inc.
Geotechnical Engineer – David Blackmore and Associates

The Buidling:

Size: 111,570 S.F.
Stories Above Grade: 6
Cost : $21.5 Million Dollars
Building Completion: October 2007
Occupancy: The Building Is A
Mixed Use Developement
Containing Retail and
Housing
Delivery Method: Design-Bid -Build

Architecture:

"Modern" and "Sophisticated" Design
Combines the uses of Metal, Glass,
Stucco And Masonry

Lighting\Electrical:

Primary Fluorescent Lighting
208/120 3 Phase, 4 Wire Main Sytem
Emergency Power: Diesel Generator
120/208V, 24hrs Capacity

Structural:

Foundation: 6" SOG With WWR Reinfrocing,
Drilled Piers
SuperStructure: Structural Steel With Masonry
And Steel Stud Load Bearing Walls
Floor System: 10" Precast Planks
Roof System: Steel Joists Supporting Steel Deck,
Ruber Membrane And Rigid Insulation

Mechanical:

Main Components:
Engineered Air Make-Up Air RoofTop Units
3500 CFM, Cooling - 293,000 BTU
Heating - 341,000 BTU
GC Zoneline Terminal AC Units
310CFM, Cooling - 14,300 BTU
Heating 13,200 BTU

Design Consideration: Site Location Existing Above SEPTA Commuter Rail Tunnel

Joseph G. Lichman Jr. - Structural Option

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Executive Summary

Pearl Condominiums is located on 9th and Arch Street in Philadelphia, Pennsylvania. This structure is a mixed use development building. The building includes a retail floor at the ground level containing 10 units and five floors of housing above containing a total of 90 condominium units. One of the main design considerations for the site was the location of an existing SEPTA commuter rail tunnel which runs under the site

The objective of this report is to compare the existing structure of Pearl Condominiums building as a hybrid type of construction which uses load bearing wall and precast concrete planks to the redesigned structure which implements the use of the Flex Frame system that uses the combination of girder slab technology. The major factor influencing the redesign is the elimination of interior load bearing walls, thus resulting in a more flexible floor plan. The lateral system will also be changed to concrete shears walls to replace the concrete masonry shear walls present in the existing design.

The second topic that will be discussed in the paper is the foundation system. The analysis will focus on a possible alternative to the use of drilled piers, but will also include the effect on the train tunnel that runs underneath the site. This will help to determine if drilled piers were the most economical system for the foundation of Pearl Condominiums.

The paper will also briefly discuss two breadth topics concerning construction management and sustainability. It is proposed to analyze the effect of the Flex Frame system on construction cost and scheduling. Proposed for the issue of sustainability is the certification of the building for a LEED rating and how this will affect the building redesign.

Existing Gravity and Lateral Systems:

The gravity system of this building is comprised of load bearing walls and precast concrete planks. The main component in the lateral system is the use of concrete masonry units as shear walls in the stair towers and the elevator core. The ground floor contains moment frames to transfer lateral loads from the stair tower shear walls which end on the second floor. Finally, metal stud walls with metal strapping are used to help resist lateral load in the east to west direction of the building. From research and analysis performed during this semester the existing structural system used in Pearl Condominiums, was a very efficient and economical construction type to use for this type of building.

Conclusion:

After completing the analysis of the Flex Frame system, the conclusion is that this system is a viable solution to replace the typical load bearing wall type of construction. The cost and the time of construction required is not that much different for this project therefore that is not a factor to limit the use of the Flex Frame system. This system is only limited by the spans of the 8" precast planks because they are the only one the Flex Frame system uses. Overall both systems are very well designed for this type of construction.

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Acknowledgements

I would like to take this opportunity to acknowledge and thank all the people, design professionals and firms that gave up their time and effort to help in the completion of this senior thesis:

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Mr. Bob Hicks

Blackney Hayes Architects
Mr. Michael Resnic

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Mr. Keith Weitknecht
Mr. Mike Padula

The Pennsylvania State University
Dr. Linda Hanagan
The entire AE faculty and staff

Special thanks extended to my family and friends that have supported me through these last five years.

Building Background and Project Information

Pearl Condominiums is a mixed use development housing including 10 retail units on the ground floor and 90 condominium units on the upper floors. The gross floor area is 111,570 square feet and has 6 stories above grade. The start of construction was March 30, 2006 and the finish date is October 2007. The zoning is C-4 Commercial. Design considerations for the site included the site location existing above a SEPTA commuter rail tunnel. The Project Delivery method used was Design-bid-build and the total cost of the project was $22,646,674.

Primary project team:

- Owner - Parkway Corporation
- Architect – Blackney Hayes Architects
- Construction Manager – JJ Deluca Company Inc.
- Structural Engineer – Pennoni Associates Inc.
- Civil Engineer – Valimer Associates, LLP
- Mechanical Engineer – M.P. Hershman, PE, Inc.
- Electrical Engineer – DGW Electrical Engineers, Inc.
- Geotechnical Engineer – David Blackmore and Associates
- Plumbing and Fire Protection – Pan Am Consulting, Inc.

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Existing Structural System

Foundations:

The primary support for the foundation is the use of drilled piers. The drilled pier option was performed, so the loads from the building would be transferred from the pier to the soil below the SEPTA commuter train tunnel. The drilled piers range in size of diameter from 3'-0" to 3'-6" to 4'-0 To help distribute the load to the drilled piers the use of grade beams was employed. They range in width from 12" to 40" and in depth from 18" to 30". The slab on grade is 6" reinforced with 6x6 W2.9xW2.9 WWR over 6" crushed stone over 6 mil. Vapor retarder. (See Figure1)
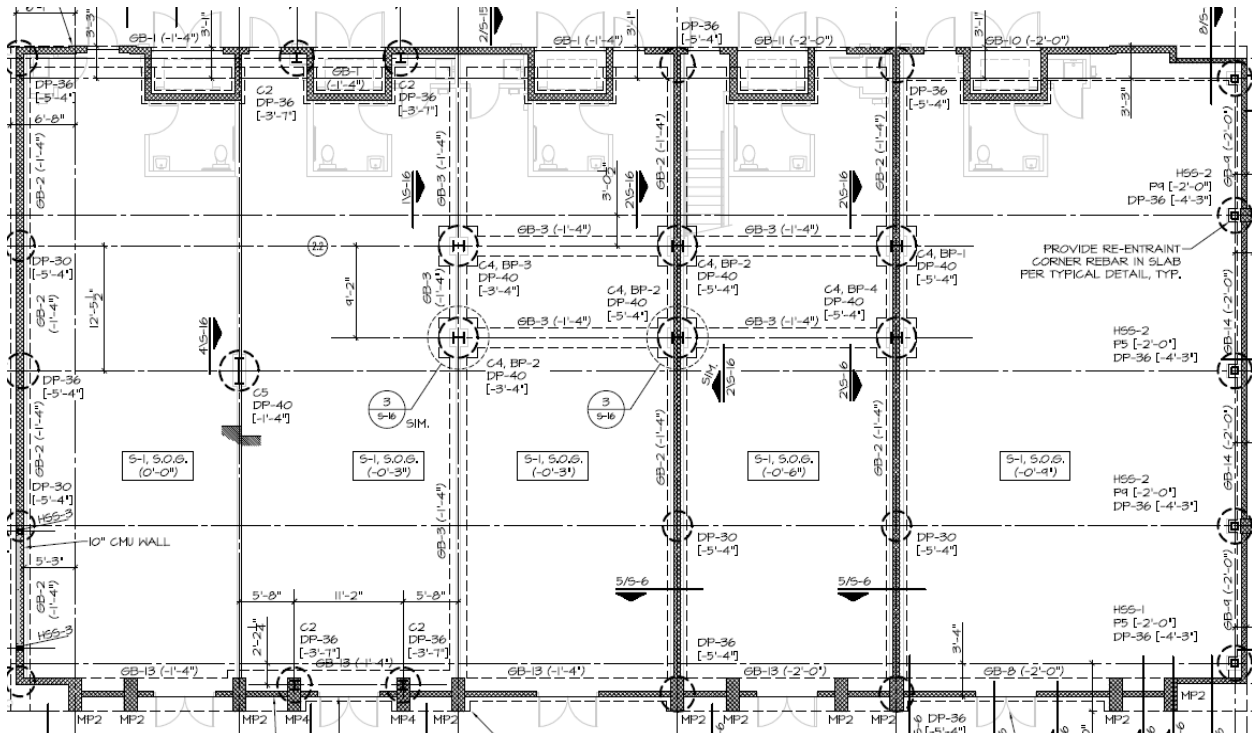

Figure 1 – South Side of Building Foundation Plan

Columns \ Load Bearing Walls:

The columns in Pearl Condominiums are used in two different types of loading. The HSS columns are used to take gravity loads, which occur at the ends of the building to support the precast concrete planks (Figure 2) and the Wide flange columns are used to resist lateral loads which occur on the ground floor in the moment frames (Figure 3)
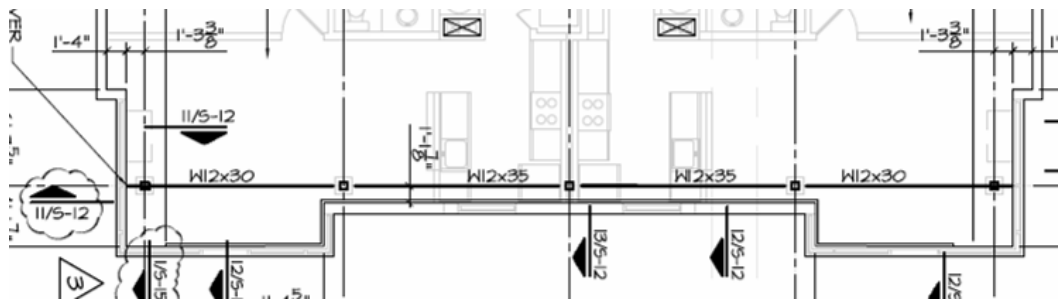

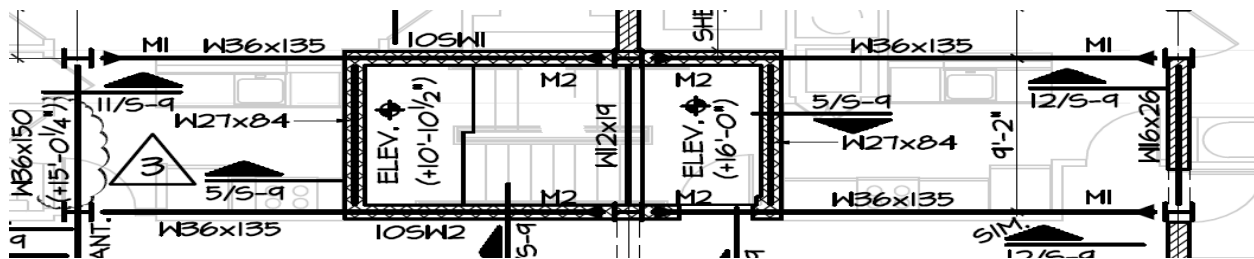Figure 2 – HSS Columns at the south end of the building

Figure 3 –Wide Flange Columns at the south side of the building

The interior bearing walls are comprised of 8 inch metal studs that are spaced at 12 inches and 16 inches on center depending on the floor location of the wall. (See Figure 4)

LOAD BEARING METAL STUD WALL SCHEDULE		
	LOAD BEARING TYPE 1 STUD WALL	
LEVEL	SIGMA STUDS	MARINOWARE STUDS
7TH – RF.	800S6300-33 @ 16"	800S200-33 @ 16"
6TH – 7TH	800S6300-33 @ 16"	800S200-54 @ 12"
5TH – 6TH	800S6300-54 @ 16"	(2) 800S200-54 @ 12"
3RD – 5TH	800S6300-68 @ 16"	(2) 800S200-54 @ 12"
2ND – 3RD	800S6300-68 @ 12"	(2) 800S200-68 @ 12"
LEVEL	LOAD BEARING TYPE 2 STUD WALL	
7TH – RF.	800S6300-33 @ 16"	800S200-33 @ 16"
6TH – 7TH	800S6300-33 @ 16"	(2) 800S200-43 @ 12"
5TH – 6TH	800S6300-54 @ 16"	(2) 800S200-54 @ 12"
3RD – 5TH	800S6300-68 @ 16"	(2) 800S200-68 @ 12"
2ND – 3RD	800S6300-68 @ 12"	(2) 800S200-47 @ 12"
LEVEL	LOAD BEARING TYPE 3 STUD WALL	
7TH – RF.	800S6300-33 @ 16"	800S200-33 @ 16"
6TH – 7TH	800S6300-33 @ 16"	(2) 800S200-54 @ 12"
5TH – 6TH	800S6300-54 @ 16"	(2) 800S200-54 @ 12"
3RD – 5TH	800S6300-68 @ 16"	(2) 800S200-68 @ 12"
2ND – 3RD	800S6300-68 @ 12"	(2) 800S200-47 @ 12"

Figure 4 – Metal Stud Bearing Wall Schedule

Floor System:

The floor system for level 2 thru 6 is comprised of a 10" Precast Concrete Plank with a ¾" concrete thick topping. (See Figure 5) The concrete strength of the precast plank is f'c equals 5,000 psi.

Grouted weight of slabs is 66 lbs. per sq. ft.

f'c = 5,000 psi   f'ci = 3,500 psi   Area = 401 in.²
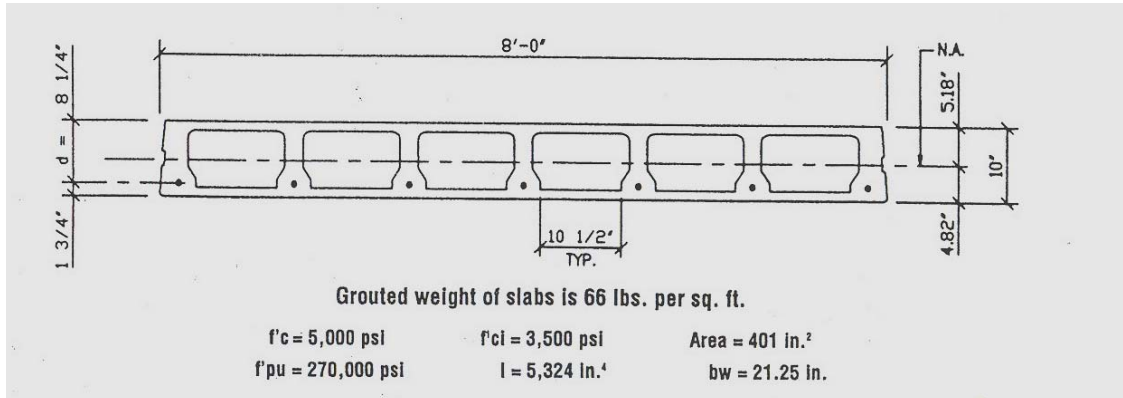f'pu = 270,000 psi   I = 5,324 in.⁴   bw = 21.25 in.

Figure 5 – Section Properties of Precast Planks

Level two acts as a transfer level, which requires the use of wide flange beam (W36). These transfer beams eliminate the need for load bearing walls to distribute the load to the foundations. The result of having these elements increases the available floor area for the retail units. (See Figure 6)
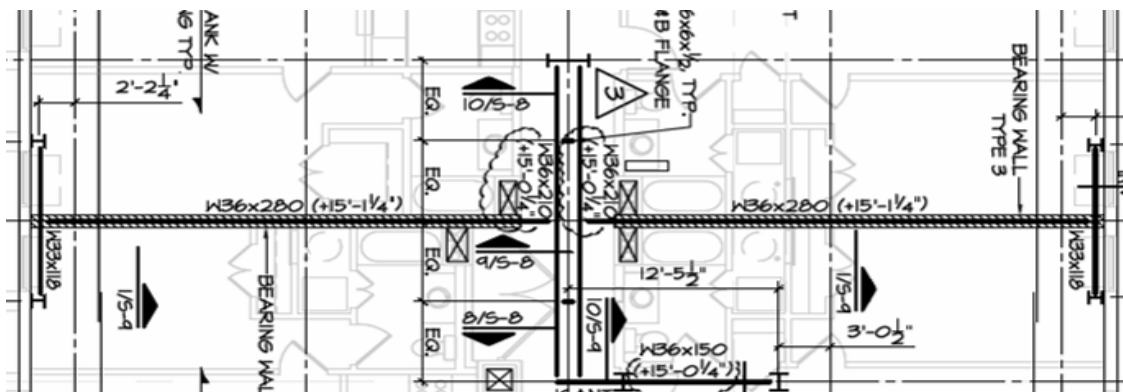

Figure 6 – Transfer Beams located on the Second Floor.

Typical Floor Plan

As shown in Figure 7, Pearl Condominiums shape and layout of the space is symmetric. This lends itself to the idea of using a structural system that is uses repetition in the design of the floor system. The floor system consisting of precast planks and metal stud bearing walls can easily duplicated from floor to floor.

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Lateral Resisting System:

The Lateral System in the building is comprised of three types: concrete masonry unit (cmu) shear walls, moment frames and metal stud shear wall. (See Figure 7)

The concrete masonry unit shear walls are used around the elevator and stairway towers. These walls range from thickness of 10" in the stair areas and 12" in the elevators. The strength of the concrete masonry units (f'm) range from 1500 psi to 2000psi and 3000psi depending on the area they are used in.

The stair tower cmu walls end on the second floor, which results in the use of moment frames on the first floor to transfer the loads from the cmu shear walls on the second floor to the foundation below.

The metal stud shear walls are composed of 8" metal studs varying in thickness. The two heights of the studs are 13'-8" and 9'-0". Metal diagonal straps connected by #12 screws to the metal studs and 7/8" diameter anchor bolts connected through different boot types help to resist the lateral forces applied to the metal studs. The metal studs are covered by gypsum wall board.
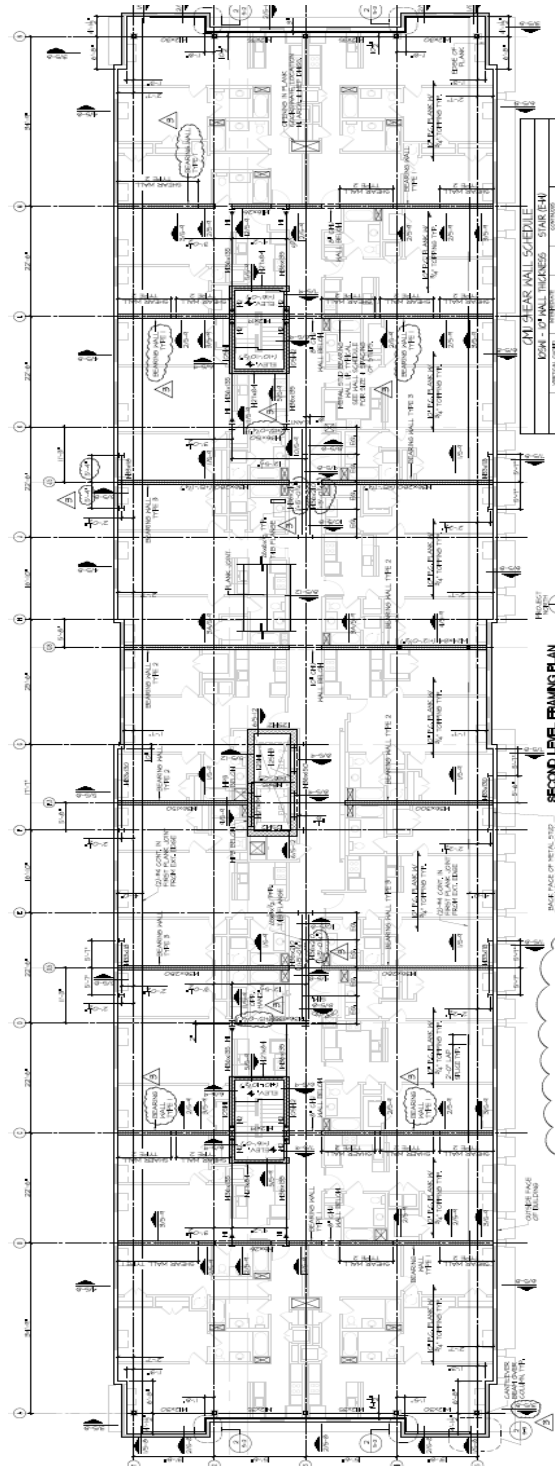

Figure 7 –Lateral Resisting System Present in Pearl Condominium

Problem Statement

During the analysis of the gravity and the wind force resisting systems present in Pearl Condominiums, it is clear that the systems chosen by the design team is currently the most efficient possible. A possible issue arose during the analysis of the gravity system with the use of the load bearing wall, which limited the flexibility of the floor plans. For the redesign the floor system, the main objective will be to try and see if another structural system is possible to be able to create a more flexible floor plan by eliminating the need for the interior load bearing walls. Another aspect that will be investigated will be the lateral shear walls changing the material from concrete masonry units to reinforced concrete With respect to the foundation system, the objective will be to compare other possible systems to the existing drilled piers system to see if any are a viable alternative. This will prove if the current system was the most economical and efficient system used because of the impact of the SEPTA commuter train tunnel that runs underneath the site.

Problem Solution

Floor system:

The redesign of the system includes first and foremost the removal of the interior load bearing walls. To compensate for the loss of the interior load bearing walls, the proposed structural system to be implemented is the Flex Frame system. This floor system is comprised of precast concrete planks which are supported by special steel "d beams". The precast planks are grouted solid around the "d beam" to create the beam to plank connection. (See Figure 3) The "d beam" is a specialty beam which is created by cutting a wide flange beam in half and adding a plate, with a smaller width than the bottom flange, to create a top flange.

With the Flex Frame system there are a few limiting factors such as the deepest member that is currently available is a nine inch "d beam", this will decrease the possible span and tributary width that the beam can carry. From this the spans of the precast concrete planks will be reduced, in turn reducing the size of the typical bay. The overall geometry of the building will not be changed but the implementation of a column grid will be used to facilitate in the redesign. This column grid will be beneficial in seeing the impact of the new system on the layout of the space on the floor plans.
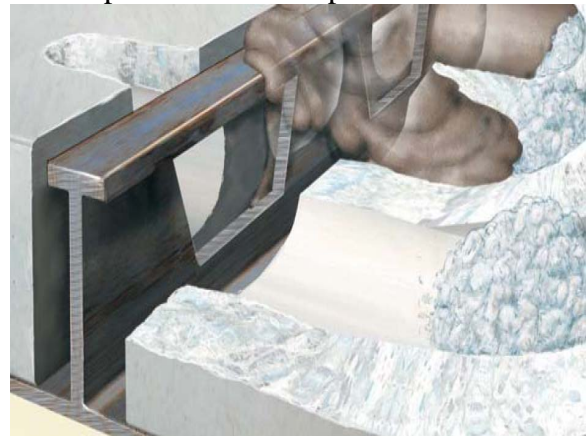
Structural Depth Study

The objective for the depth study is the analysis of the Flex Frame system in replacing the load bearing walls used in Pearl Condominiums. This is the result of trying to create an open floor plan by eliminating the use of the load bearing walls.

The course of action that was taken for the design was to start at the roof and work the way down the building. The existing roof framing was steel joists. The steel joists were changed into steel beams and girders to eliminate the need for the bearing walls. With the removal of the walls the implementation of the HSS columns were required to pick up the load from the steel girders. The roof load was added to by the addition of the extensive green roof, in an effort to gain points for a LEED certification. The total load applied to the steel beam was 45 psf for dead and 30 psf for live load. The grid for the entire structural system was developed to limit the interference on the architectural plans. The columns followed the load path that the load bearing walls were designed. For the roof the largest span was 34'-9", which resulted in a beam the size of W14x 61. The columns were spaced at 11'-8" on the column line. (See Figure 8 for typical framing)
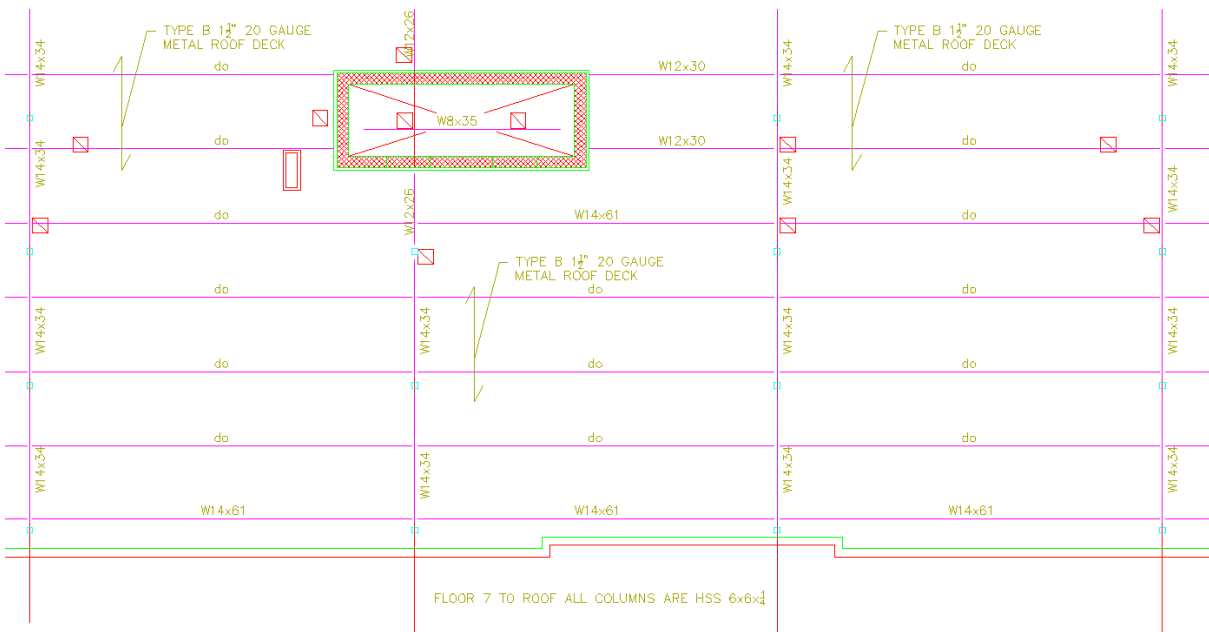

Figure 8 – Roof Framing

For Floor 3, 5, 6, and 7 the framing was all the same. This is were the implementation of the Flex Frame Occurred The Dissymmetric beam that was chosen for this design was the DB 9x46, this is the largest size that is currently available. The limitation to the Flex Frame system that arose during the redesign is in the use of the precast planks. The only precast plank that can be used is the 8" thick precast plank. For the required spans that are present in this design the 8" precast plank with 2" topping will have to have a 1" camber before erection to limit the deflection after the load is applied. (See Figure 9 – 4HC8+2; 78S span 33'-34' chosen for design)
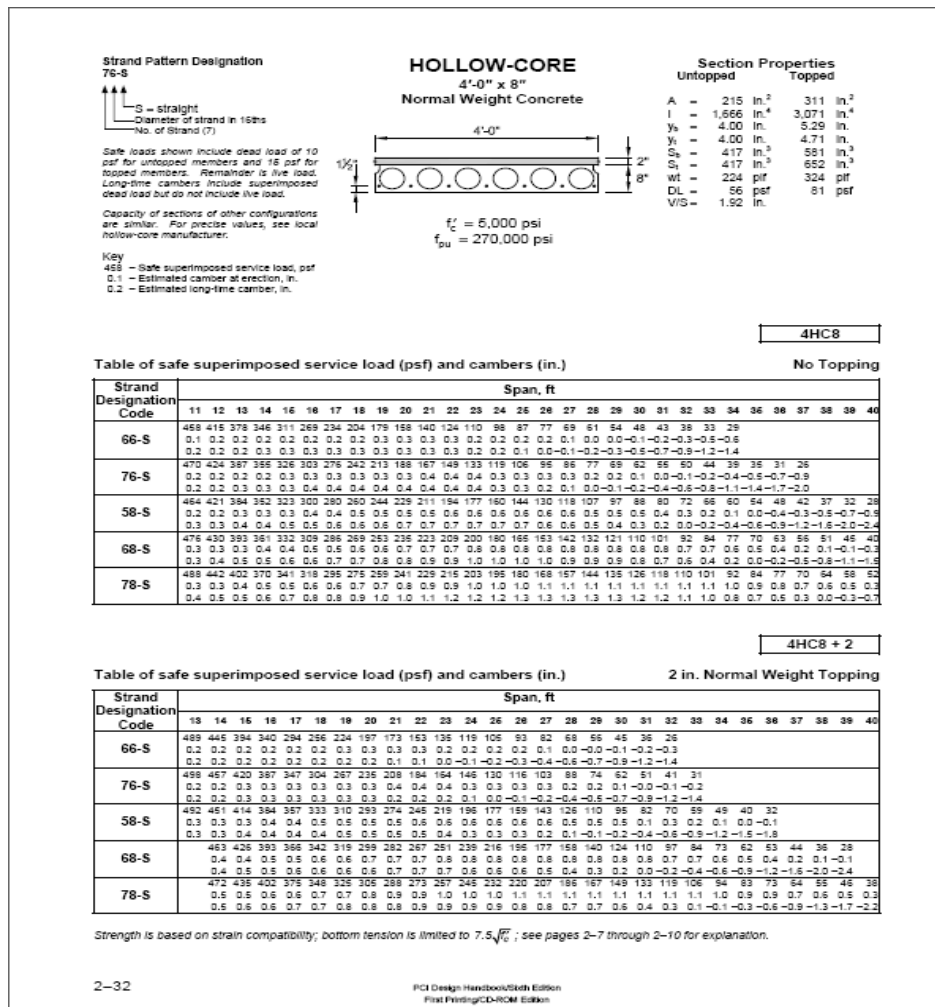
Figure 9 – Precast Plank

D-Beam® Properties Table
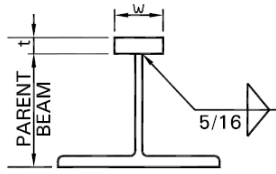
| Designation	Steel Only / Web Ignored						Transformed Section / Web Ignored				
	Ix	C bot	C top	S bot	S top	Allowable Moment Fy=50 KSI f$_b$=0.6 Fy	Ix	C bot	C top	S bot	S top
	in⁴	in	in	in³	in³	kft	in⁴	in	in	in³	in³
DB 8 x 35	102	2.80	5.20	36.5	19.7	49	279	4.16	4.40	67.1	63.5
DB 8 x 37	103	2.76	5.24	37.3	19.7	49	282	4.16	4.42	67.7	63.8
DB 8 x 40	122	3.39	4.61	36.1	26.5	66	289	4.26	4.30	67.9	67.2
DB 8 x 42	123	3.35	4.65	36.9	26.5	66	291	4.26	4.32	68.4	67.5
DB 9 x 41	159	3.12	6.51	51.0	24.4	61	332	4.27	5.35	77.7	62.1
DB 9 x 46	195	3.84	5.79	50.8	33.7	84	356	4.43	5.20	80.6	68.6

Figure 10 - D-Beam

The D-Beam chosen is DB9x46 (See Figure 10 for properties). The maximum span of this beam is 13' with the tributary width of 34'.(For calculations see Appendix) The D-Beams help to decrease the required depth of the floor system compared to wide flange beam system. The construction process for the girder slab system incorporates the use of grout which creates a composite action for the D-Beam and the precast planks increasing the strength which is also an advantage over the traditional load bearing walls with precast planks. The girders at the end of the building on Column lines A & N are the opposite; they are placed below the placed below the precast planks to support them. (See Figure 11 for typical floor framing)
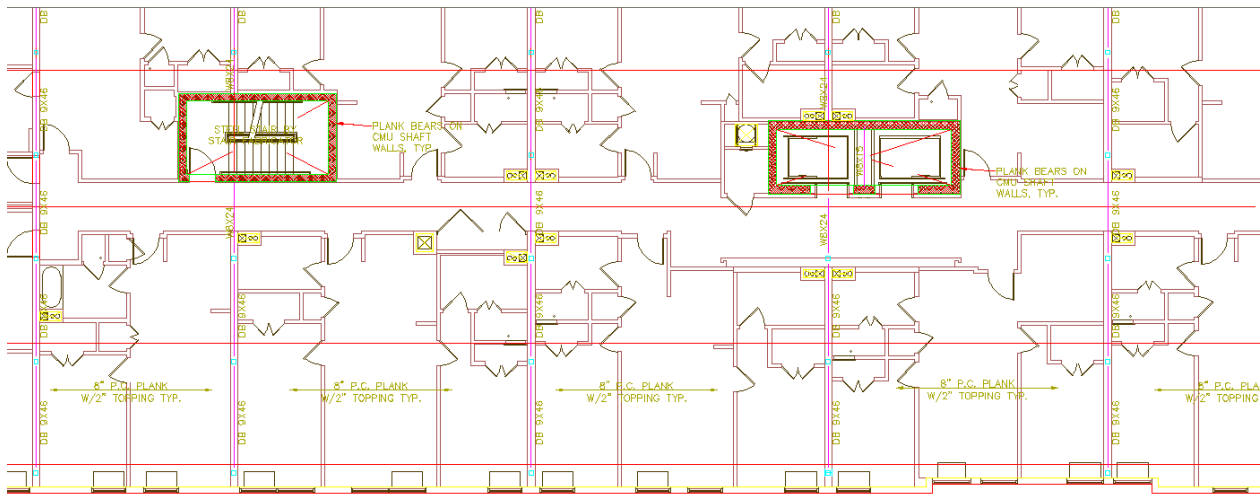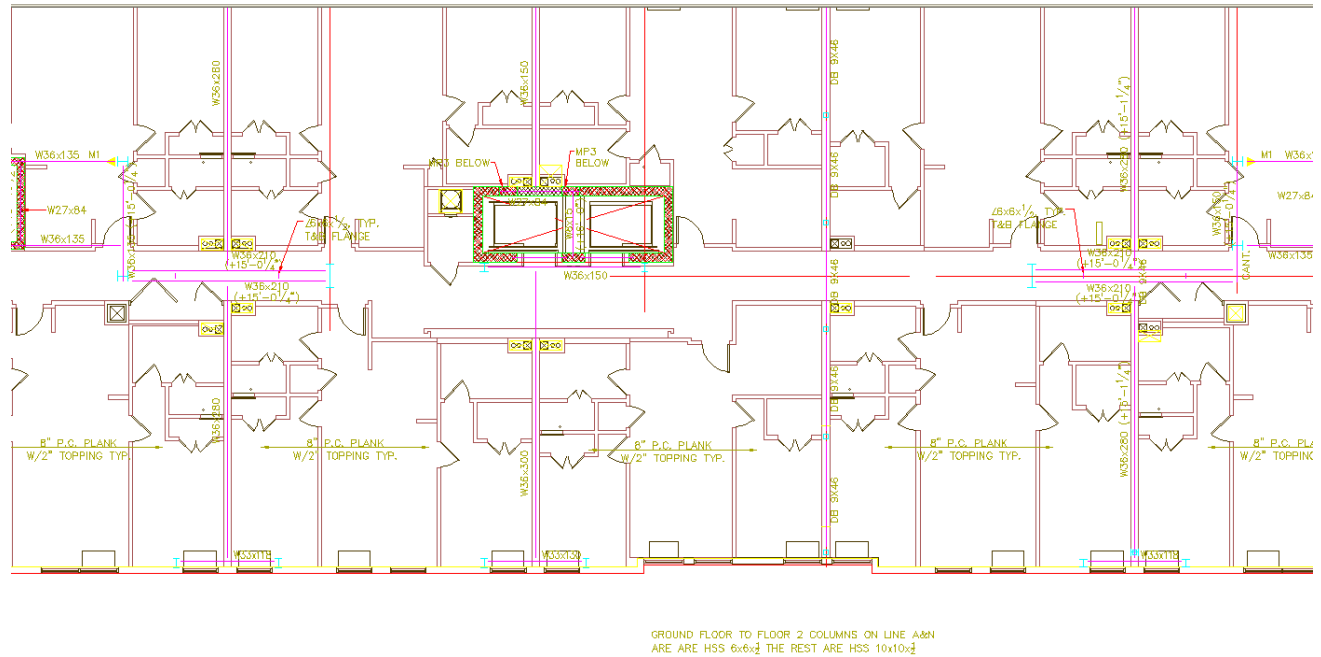

Figure 11 - Typical Floor Framing

Figure 12 – Second Floor Framing

The second floor framing remains the same because of trying to keep the same architectural floor plans that are now in the existing system. The major change is the removal of the masonry load bearing walls and now giving the possibility of expanding the retail space into another without having to worry about making changes to the walls. The plans are now more open.

The change from the concrete masonry shear walls to reinforced concrete shear walls did not affect the building structural design. With respect to the lateral system the size of the concrete wall required was only 8" with both horizontal and vertical reinforcement of #5 at 14" on center. The only major change is in the reduction of the width of the walls from 12" to 8". (For calculations see Appendix)

With respect to the foundation system only one row of columns would be possible to have unrestricted foundation construction, which would allow for the use of piles without needing to reinforce the tunnel. To compare prices for the pile using HP 10x42 the cost per linear foot is $28 and the price for a drilled pier of 10" diameter is $18 per foot. The drilled pier for this project was the most economical and most efficient way. This type of construction of using the drilled pier creates the least amount of disturbance to the tunnel below and eliminates the need for reinforcing the tunnel, which would have greatly increased the cost of the project.

Breadth Study #1: Construction Cost and Schedule Analysis

The goal of this breadth study was to compare the existing structure of Pearl Condominiums to the redesign Flex Frame System. The analysis consisted of the structural system that started at the columns of the ground floor. The decision was made not to make any changes to the foundation system and analyze the impact of the new system on the floors above the ground floor. The use of RS Means Building Construction Data Cost book facilitated the estimation of the cost and time for construction for the new system.

Starting the comparison, the base values of the existing load bearing wall system are 3 months for construction and the cost of construction was $1,754,524. The construction being compared is the precast planks, the load bearing walls that support them, the transfer level on the second floor and the roof framing.

The redesign using the Flex Frame resulted in the time of construction of 2 months and 12 days (See Figure for schedule), along with the cost of construction reacting $1,760,136. The cost per floor and columns is as follows:

Roof – $232,011.37
Column from Roof to Level 7 – $24,510
Floor 7 - $206,167.20
Columns from Level 6 to 7 - $24,510
Floor 6 - $206,167.20
Columns from Level 5 to 6 - $24.510
Floor 5 - $206,167.20
Columns from Level 3 to 5 - $24,510
Floor 3 - $ 206,167.20
Columns from Level 2 to 3 - $43,075
Floor 2 - $327,867.53
Columns Ground to Level 2 - $342,492
Concrete Walls for Elevator and Stair Cores - $191,981

Conclusion:
After comparing the cost and time for construction, the Flex Frame system is a viable system to replace the load bearing wall and precast plank system. The difference in the time of construction and the cost is small enough that either system is a possible solution for this building. With the Flex Frame system the open-web dissymmetric beam has a added cost of $1.25 per pound for the added fabrication required to produce this beam.
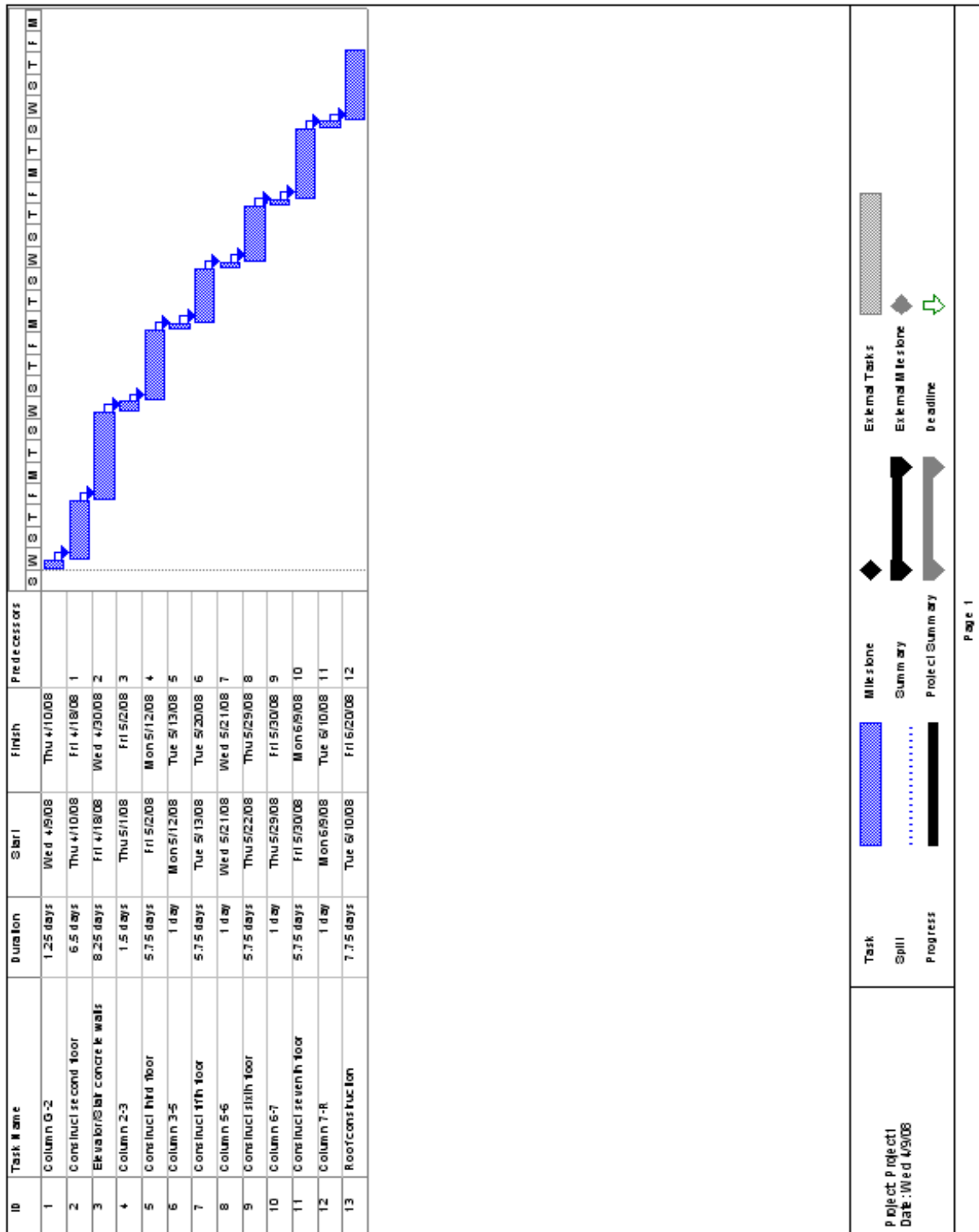
ID	Task Name	Duration	Start	Finish	Predecessors
1	Column G-2	1.25 days	Wed 4/9/08	Thu 4/10/08	
2	Construct second floor	6.5 days	Thu 4/10/08	Fri 4/18/08	1
3	Elevator/Stair concrete walls	8.25 days	Fri 4/18/08	Wed 4/30/08	2
4	Column 2-3	1.5 days	Thu 5/1/08	Fri 5/2/08	3
5	Construct third floor	5.75 days	Fri 5/2/08	Mon 5/12/08	4
6	Column 3-5	1 day	Mon 5/12/08	Tue 5/13/08	5
7	Construct fifth floor	5.75 days	Tue 5/13/08	Tue 5/20/08	6
8	Column 5-6	1 day	Wed 5/21/08	Wed 5/21/08	7
9	Construct sixth floor	5.75 days	Thu 5/22/08	Thu 5/29/08	8
10	Column 6-7	1 day	Thu 5/29/08	Fri 5/30/08	9
11	Construct seventh floor	5.75 days	Fri 5/30/08	Mon 6/9/08	10
12	Column 7-R	1 day	Mon 6/9/08	Tue 6/10/08	11
13	Roof construction	7.75 days	Tue 6/10/08	Fri 6/20/08	12

Project: Project1
Date: Wed 4/9/08

Task		Milestone	◆	External Tasks	
Split		Summary		External Milestone	◆
Progress		Project Summary		Deadline	⇨

Page 1

Figure - Schedule for Flex Frame System

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Breadth Study #2: LEED Certification Study

The objective for this breadth study is to be able to gain a LEED certification for this building. When this building was in design the idea of LEED was still new, so it was not incorporated into the design. Some of the Possible Points to gain for this project are:

SS 4.1 Alternative Transportation – The SEPTA commuter rail tunnel is less than a 1/2 mile away.

EQ Prerequisite 2: Environmental Tobacco Smoke (ETS) Control -Prohibit smoking in the building. & locate any exterior designated smoking areas at least 25 feet away from entries, outdoor air intakes and operable
windows.

SS Credit 7.2: Heat Island Effect: Roof -Install a vegetated roof for at least 50% of the roof area

There are also ways to gain credits by implementing low-flush toilets, the tenants of the units have control of their thermostat. The only part that is regulated temperature is by the owner are the hallways and the common areas.

It is possible for this building to become LEED certified if during the design process, it is harder to go back in after the building is constructed to achieve a LEED certification.

Summary and Conclusion

After completing the analysis of the Flex Frame system, the conclusion is that this system is a viable solution to replace the typical load bearing wall type of construction. The cost and the time of construction required is not that much different for this project therefore that is not a factor to limit the use of the Flex Frame system. This system is only limited by the spans of the 8" precast planks because they are the only one the Flex Frame system uses. Overall both systems are very well designed for this type of construction.

Appendix

Floor Live Loads	
Occupancy or Use	**Uniform Live Load (psf)**
Condominium Units w\ Partitions	60
Retail Units (first floor)	100
Stairs	100
Corridor above first floor	80
Corridor at first floor	100
Roof	30

Floor Dead Loads	
Occupancy or Use	**Uniform Dead Load (psf)**
Concrete Precast Plank	66
Roof	20

Superimposed Floor Dead Loads	
Occupancy or Use	**Uniform Dead Load (psf)**
Roof	20
Condominium Units w\ Partitions	25
Corridor above first floor	25
Corridor at first floor	25
Retail Units	25

Snow Loading	
Item	**Value**
Ground Snow Load (Pg)	25 psf
Exposure Factor	B
Roof Exposure	Fully Exposed
Exposure Factor (Ce)	0.9
Thermal Factor (Ct)	1.0
Occupancy Category	II
Importance Factor (Is)	1.0
Flat-Roof Snow Load $P_f = 0.7 C_e C_t I_s P_g$	16 psf

Wind Loads:

North to South Direction

Windward Calculations							
Level	z	Kz	qz	Cp	Ext. Pressure	GCpi	P_{total} (+GC_{pi}) (psf)
1	0	0.57	10.05	0.80	6.83	+/- 0.55	-1.89
2	16	0.62	10.93	0.80	7.43	+/- 0.55	-1.29
3	25.92	0.70	12.34	0.80	8.39	+/- 0.55	-0.33
4	35.83	0.76	13.40	0.80	9.11	+/- 0.55	0.38
5	45.75	0.81	14.28	0.80	9.71	+/- 0.55	0.98
6	55.67	0.85	14.98	0.80	10.19	+/- 0.55	1.46
Roof	72.3	0.90	15.86	0.80	10.79	+/- 0.55	2.06

Leeward Calculations						
Level	z	Kz	q_h (psf)	C_p	External Pressure	P_{total} (+GC_{pi}) (psf)
1	0	0.57	15.86	-0.20	-2.70	-11.42
2	16	0.62	15.86	-0.20	-2.70	-11.42
3	25.92	0.70	15.86	-0.20	-2.70	-11.42
4	35.83	0.76	15.86	-0.20	-2.70	-11.42
5	45.75	0.81	15.86	-0.20	-2.70	-11.42
6	55.67	0.85	15.86	-0.20	-2.70	-11.42
Roof	72.3	0.90	15.86	-0.20	-2.70	-11.42

Negative Internal Pressure		
q_h (psf)	GC_{pi}	P_{neg} (psf)
15.86	-0.55	-8.7
Positive Internal Pressure		
q_z (psf)	GC_{pi}	P_{pos} (psf)
15.86	0.55	8.7

Total		
Level	P_{total} (+GC_{pi}) (psf)	Force at Floor (kips)
1	9.53	0
2	10.13	52
3	11.09	43
4	11.81	45
5	12.40	46
6	12.88	64
Roof	13.48	41

Seismic Loads:

Occupancy Category – II
Importance Factor – 1.0
Seismic Design Category – B
Response Modification Factor – 5.5
    (Reinforced Masonry Shear Walls)
Site Class - D
$S_s = 0.270g$
$S_1 = 0.060g$
$F_a = 1.585$
$F_v = 2.4$
$S_{DS} = 0.287$
$S_{D1} = 0.096$

Seismic Base Shear:
    $V = Cs*W$

    $W = 11796\ k$
    $Cs = 0.0285$
        $R = 4$ (Reinforced Concrete Shear Wall)
    $V = 336k$

Vertical Distribution of Forces:
    Fundamental Period:
        $Ta = 0.496\ sec$
        $K = 1.0$

Level	wx	hx	wx*hx^k	Cvx	Fx	Mx
2	2171	16.000	34736	0.0817	32.3	516.8
3	2080	25.917	53907.36	0.127	42.7	1106.7
4	2064	35.833	73959.312	0.174	58.5	2096.2
5	2064	45.750	94428	0.222	74	3385.5
6	2115	55.667	117735.705	0.277	93.1	5182.6
Roof	736.26	68.500	50434	0.119	40	2740
				Overturning Moment =		15028

Roof Structural System

- Loads
  - Green Roof
    - extensive type — 25 psf
  - Dead Load — 20 psf
  - Live Load — 30 psf

  - Total Load $(25 + 20 + 30) = 75$ psf

Max Beam Span — 35´

Roof Metal Deck
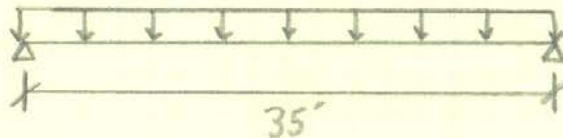 - Type B $1\frac{1}{2}"$ — 20 GA
  - Double Span Condition
  - Span — $6´-6"$ 81 psf max uniform total load

Worst Case Beam

Span — 35´

Tributary width — $6´-6"$

$w = (75\,psf)(6.5´) = 487.5$ plf

35´

$M_q = \dfrac{0.488\ \text{klf}\ (35)^2}{8} = 74.73\ \text{K-ft}$ $V = 0.488(35/2)$
$= 8.54\ \text{K}$

$S_{x\,req} = \dfrac{(74.73\ \text{K-ft})(12\,in/ft)}{(0.66)(50\,ksi)} = 27.17\,in^3$

Try 14×61

$F_{cr} = \dfrac{1.14(\pi^2)\ E}{\left(\dfrac{35(12)}{2.78}\right)^2}\sqrt{1 + 0.078\left(\dfrac{2.19(1)}{92.1\,(13.2)}\right)\left(\dfrac{35(12)}{2.78}\right)^2}$

$= 29.3\ ksi$

Try W12×26

$$M_a = \frac{1.65 \, klf \, (11.67)^2}{8} = 28.1 \; k\text{-}ft$$

$$L_b = 11'\text{-}8'' > Lp = 5.33$$

$$\emptyset M_n = 1.14 [92.8 - 3.61(11.67 - 5.33)]$$

$$= 79.7 \; k\text{-}ft > 28.1 \; k\text{-}ft$$

Girder by Elevator

span - 8'-8"

tributary width - 35'

$$w = 35' (75 \, psf + 9.38 \, psf) = 2.95 \, klf$$
$$\uparrow \, 61 plf / 6.5'$$

$$M_a = \frac{2.95 \, klf \, (8.67')^2}{8} = 27.7 \; k\text{-}ft$$

Use W12×26

Level 7 Columns

Worst Case
Tributary Area

$$35' \times 12'\text{-}2'' = 426 \, sq\,ft$$

$$P_u = 426 \, sq\,ft \, (75\,psf + 10\,psf) = 36.21 \, k$$
$$\uparrow \text{steel beam / girder}$$

Unbraced Length - 10'

Use HSS 6×6×¼ for all Columns

Steel Manual Table 4-4

$$KL = 10' \quad P_n / \Omega_c = 121^k > 36.21^k$$

7th Floor

Loads

- Dead
  - 60 psf - 8" precast plank w/ 1" camber
    + 2" topping (add 25 psf)

- Live Load
  - 40 psf (typical room)
  - 20 psf (partition Loading)
  - 80 psf (corridors)

Open-Web Dissymmetric beam (D-Beam)

Worst Case

Dead Load - 60 psf

Live Load - 80 psf

Topping Load - 25 psf

DB span - 13 ft

Plank span - 34 ft

Grout f'_c - 4000 psi

Allowable $\Delta_{LL} = L/360$

Allowable $\Delta_{LL} = 0.43$ in

Initial Load - Pre Composite

$$M_{DL} = (34 ft)(0.06 \, ksf)(13 ft)^2/8$$

$$= 43.1 \, ft\text{-}k < 84.0 \, ft\text{-}k$$

$$\therefore \; O.K.$$

DB Properties

DB 9 x 46

Steel Section
I_s - 195 in^4

S_t - 33.7 in^3

S_b - 50.8 in^3

M_{scap} - 84.0 ft-k

t_w - 0.375 in

b - 5.75 in

Transformed Section
I_t - 356 in^4

S_t - 68.6 in^3

S_b - 80.6 in^3

$$\Delta_{DL} = \frac{5(34ft)(0.06\ ksf)(13ft)^{4}(1728\ in^3/ft^3)}{(384)(29000ksi)(195\ in^4)} = 0.23\ in$$

$\Delta\ Ratio = L/673$

$$13\times12/0.23 \approx 673$$

Total Load - Composite

$M_{sup} = (34ft)(0.08 + 0.025\ ksf)(13ft)^2/8 = 75.4\ ft\text{-}k$

$M_{TL} = 43.1\ ft\text{-}k + 75.4\ ft\text{-}k = 118.5\ ft\text{-}k$

$S_{req} = (118.5\ ft\text{-}k)(12\ in/ft)/(0.60)(50\ ksi) = 47.4\ in^3 < 68.6\ in^3$

$$\Delta_{sup} = \frac{5(34ft)(0.08 + 0.025\ ksf)(13ft)^4(1728\ in^3/ft^3)}{384(29000\ ksi)(356\ in^4)} \quad \therefore O.K.$$

$= 0.22\ in \quad < 0.43\ in \quad \therefore\ O.K.$

$\Delta_{TOT} = 0.23 + 0.22 = 0.45\ in$

$= L/344$

Compressive Stress on Concrete

$N_{value} = \dfrac{E_{steel}}{E_{concrete}} = \dfrac{29,000\ ksi}{57,000(4000psi)^{1/2}} = \dfrac{29,000\ ksi}{3,605\ ksi} = 8.04$

$S_{tc} = 8.04(68.6\ in^3) = 552\ in^3$

$f_c = (75.4\ k\text{-}ft)(12\ in/ft)/(552\ in^3) = 1.64\ ksi$

$F_c = (0.45)(4\ ksi) = 1.80\ ksi > 1.64\ ksi \quad \therefore\ O.K.$

Bottom Flange Tension Stress (Total Load)

$f_b = \dfrac{(43.1\ ft\text{-}k)(12\ in/ft)}{50.8\ in^3} + \dfrac{(75.4\ ft\text{-}k)(12\ in/ft)}{80.6\ in^3}$

$= 10.18\ ksi + 11.23\ ksi = 21.4\ ksi$

$F_b = 0.9(50\ ksi) = 45\ ksi > 21.4\ ksi \quad \therefore\ O.K.$

Check Shear

Total Load = (60 + 80 + 25 psf) = 165 psf

$w = (0.165 \text{ ksf})(34 ft) = 5.61 \text{ klf}$

$R = (5.61 \text{ klf})(13 ft)/2 = 36.5 \text{ k}$

$f_v = (36.5 k)/(0.375)(5.75 in) = 16.9 \text{ ksi}$

$F_v = 0.4(50 \text{ ksi}) = 20 \text{ ksi} > 16.9 \text{ ksi}$ ∴ O.k.

Worst Case Girder By Elevator

USE DB 9×46
Same as on
Level 2,3,5,6
as on 7

Span - 8'-8"

Tributary width - 35'

$w = 35'(60 psf + 80 psf) = 4.9 \text{ klf}$

$M_a = \dfrac{4.9(8.67')^2}{8} = 46 \text{ k-ft}$ Try W8×24

Same on Levels 2,3,5,6
as on 7

$L_b = 8.67 > L_p = 5.69'$

$\varnothing M_n = 1.14[57.6 - 1.59(8.67 - 5.69)] = 60.3 \text{ k-ft} > 46 \text{ k-ft}$

Use W8×24

Girders on exterior

span - 11'-8"

Tributary width - 19.5'

$w = 19.5'(60 psf + (40 psf + 20 psf)) = 2.34 \text{ klf}$

$M_a = \dfrac{(2.34 \text{ klf})(11.67)^2}{8} = 39.8 \text{ k-ft}$ Try W12×26

$L_B = 11.67' > L_p = 5.33'$

$M_n = 1.14[92.8 \ 3.61(11.67 - 5.33)]$
$= 79.7 \text{ ft-k} > 39.8 \text{ k-ft}$

USE W12×26
Same on levels
2,3,5,6 as on 7

Columns

Level 6 - Worst Case
  Square Feet - $[35' \times 12' - 2'] = 426$ sq ft

      $P_u = 426 (85 \text{ psf}) + 426 (60 \text{ psf} + 80 \text{ psf}) = 95.85^k$
              roof   Level 7
  Unbraced length - 10'

Level 6
  End Columns

  Square Feet - $[35\text{'}/2 \times 11' - 8'] = 204.2$ sq ft

    $P_u = 204.2 (85 \text{ psf}) + 204.2 (60 \text{ psf} + 60 \text{ psf}) = 41.9^k$

  Unbraced Length - 10'

  USE HSS $6 \times 6 \times 1/4$.

              P_n/Ω_c @ 10' = 121^k Table 4-4 Steel
                                                     Manual
              $121^k > 95.85^k$

Columns
  Level 5 - 6
  Square Feet - 426 sq ft
      $P_u = 426 (85 + 2(60 + 80)) = 156^k < 173^k$
          Unbraced Length - 10'   USE HSS $6 \times 6 \times 3/8$
  End Columns   Line A & N
      $P_u = 204.2 (85 + 2(120)) = 66.4^k < 121$
          Unbraced Length - 10'   USE HSS $6 \times 6 \times 1/4$

Columns

Level 3-5

$P_u = 426(85 + 3(140)) = 216^k < 221$

Unbraced Length = 10'   USE HSS 6×6×½

Columns Line A & N

$P_u = 204.2(85 + 3(120)) = 91^k < 148^k$

Unbraced Length = 10'   USE HSS 6×6×5/16

Columns

Level 2-3

$P_u = 426(85 + 4(140)) = 275^k < 334^k$

Unbraced Length = 10'   USE HSS 8×8×½

Line A & N

$P_u = 204.2(85 + 4(120)) = 115^k < 148^k$

Unbraced Length = 10'   USE HSS 6×6×5/16

Columns

Ground - Level 2

$P_u = 426(85 + 5(140)) = 334^k < 401^k$

Unbraced length 16'   USE HSS 10×10×½
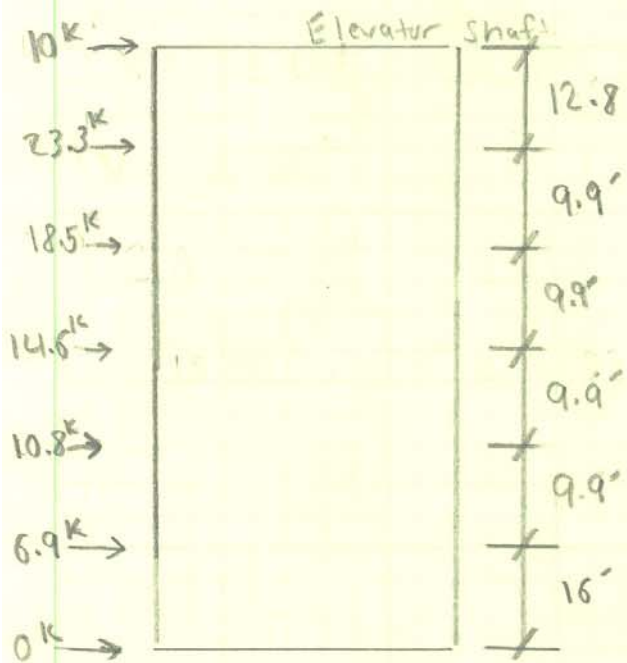
$P_u = 204.2(85 + 5(120)) = 140^k < 163^k$

USE HSS 6×6×½

Concrete Shear Wall

Story 5

East to West

Seismic Controls

Elevator Shaft

$10^k \rightarrow$

$23.3^k \rightarrow$

$18.5^k \rightarrow$

$14.6^k \rightarrow$

$10.8^k \rightarrow$

$6.9^k \rightarrow$

$0^k \rightarrow$

12.8'

9.9'

9.9'

9.9'

9.9'

16'

$$M_u = 10^k(68.5) + 23.3^k(55.67)$$
$$+ 18.5^k(45.75) + 14.6^k(35.83)$$
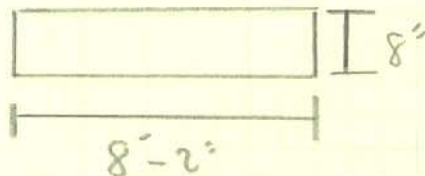$$+ 10.80(25.9) + 6.9(16)$$
$$= 3742 \text{ k-ft}$$

$V_u = 84.1^k$

$$P_u = (8'-2'')(17')(55_{psf} + 5(60_{psf}))$$
$$= 49.3^k$$

Boundary Check

$A_g = (8'-2'')(8') = 5.4 \text{ ft}^2$

$I_g = \dfrac{(8/12)(8.167)^3}{12} = 30.3 \text{ ft}^4$

$8''$

$8'-2''$

$f'_c = 5000 \text{ psi}$

$C_v = \dfrac{49.3}{2} + \dfrac{3742}{8'-2''} = 482.8 \, P_{uBE}$

$$f_c = \dfrac{49.3}{5.4} + \dfrac{3742 + \left(\frac{8.167}{2}\right)}{30.3} =$$

$$= 9.13 + 123.63 = 132.76 \text{ Ksf}$$

$$= 132.76 \text{ Ksf} = 0.92 \text{ Ksi}$$

$0.2(f'_c) = 0.2(5\text{ksi}) = 1$

$f_c = 0.92 < 1$ ∴ No Boundary Element required

Long + Transverse Reinforcement

$2 A_{cv} \sqrt{f'_c} =$

$2(8 \times 12 \times 8.167') \sqrt{5000}/1000 = 111^k > 84.1^k$ ∴ Need one Curtain

$A_{cv} = 12'' \times 8'' = 96 \text{ in}^2/\text{ft}$

$A_{s \, req} = (0.0025)(96) = 0.24 \text{ in}^2/\text{ft}$

$$\frac{0.24}{12} = \frac{0.31}{S}$$

$$S = 15.5"$$

$$S_{req} = 14" > 12" \therefore \text{o.k.}$$

check Shear

$$V_n = A_{cv}(\alpha_c \sqrt{f_c'} + \beta_t f_y)$$

$$\frac{h_w}{\ell_w} = \frac{68.5}{8.167} = 12 > 2 \therefore \alpha_c = 2.0$$

$$A_{cv} = (8)(8'-2")(12) = 784 \text{in}^2$$

$$\rho_t = \frac{0.31}{(12)(8)} = 0.0032$$

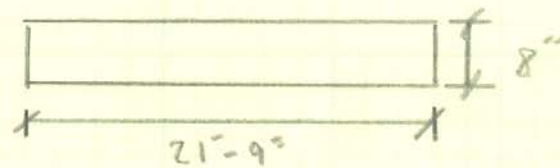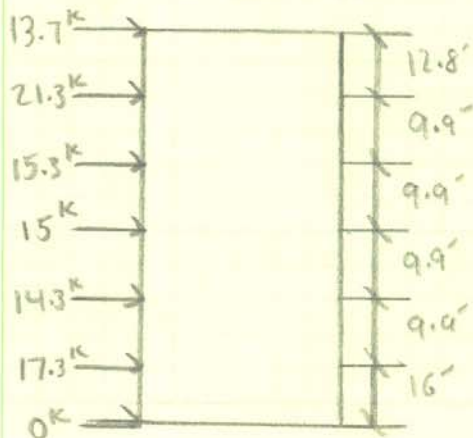$$V_n = 784 \left(2\sqrt{5000} + 0.0032(60,000)\right)/1000 = 261.4^k$$

$$\emptyset V_n = 261.4(0.6) = 156.8 > 84.1^k \therefore \text{o.k.}$$

Use 8" concrete shear wall with
#5 @ 14" each way , f'c = 5000 psi

North to South
Wind Controls

$$M_u = 13.7^k(68.5) + 21.3^k(55.67)$$
$$+ 15.3^k(45.75) + 15(35.83)$$
$$+ 14.3(25.9) + 17.3(16)$$
$$= 4009 \text{ k-ft}$$

$V_U = 97^K$

$P_U = (63.75 \text{ sqft})(55 \text{ psf} + 5(60 \text{psf})) = 22.6^K$

Boundary Check $C_v = \frac{22.6}{2} + \frac{4009}{21.75} = 195.6$
                                        P_{UBE}

$A_g = (21-9^")(8^") = 14.5 \text{ ft}^2$

$I_g = \frac{(8/12)(21.75^")^3}{12} = 571.6 \text{ ft}^4$

$f_c = \frac{22.6}{14.5} + \frac{4009 + \frac{21.75}{2}}{571.6} =$

$\quad 1.56 + 7.03 = 7.59 \text{ ksf} = 0.05 \text{ ksi} < 1.0$ for
$\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad 5 \text{ ksi}$

$\qquad\qquad\qquad\qquad\qquad\qquad\qquad \therefore$ No Boundary
Long & Transverse Reinforcement $\qquad\qquad\qquad$ Element required

$2(8)(21.75 \times 12)\sqrt{5000}/1000 = 295^K > 97^K$

$\qquad\qquad\qquad\qquad\qquad\qquad \therefore$ Need One

$A_{cv} = (12^")(8^") = 96^"$ Curtain

$\quad A_{sreq} = (0.0025)(96^") = 0.24 \text{ in}^2/\text{ft}$

$\qquad \frac{0.24 \text{ in}^2/\text{ft}}{n^"} = \frac{0.31 \text{ in}^2/\text{ft}}{S}$

$\qquad\qquad S = 15.5 \text{ in} \approx 14^" > 12^"$

Check Shear

$\qquad V_n = A_{cv}(\alpha_c \sqrt{f_c} + \rho_t f_y) \qquad \frac{hw}{\ell w} = \frac{68.5}{21.75} = 3.15 > 2$
$\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad \therefore \alpha_c = 2.0$

$V_n = 2088(2\sqrt{5000} + 0.003(60000)/1000 \quad A_{cv} = (8)(21.75 \times 12) = 2088 \text{ in}^2$
$\quad = 671^K$
$\emptyset V_n = 671^K(0.6) = 403^K > 84.1^K \qquad \rho_t = \frac{0.3)}{(12)(8)} = 0.003$